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# PROPERTIES OF CONCRETE IN AMERICAN FALLS REPLACEMENT DAM: 1- AND 5-YEAR CORE REPORT

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Concrete and Structural Branch

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16. ABSTRACT
Ten-inch-diameter cores were extracted from the concrete gravity section of American Falls Replacement Dam when the concrete was 1 year old and when the concrete was 5 years old. Tests show that the concrete was of satisfactory quality, was well compacted, and had adequate compressive strength and normal elastic properties for mass concrete. The concrete developed an inferior bond between paste and aggregate and across horizontal construction joints. The tensile strength of the concrete was also low.

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# PROPERTIES OF CONCRETE IN AMERICAN FALLS REPLACEMENT DAM: 1- AND 5-YEAR CORE REPORT

by

**Edward M. Harboe**
**(USBR Retired)**

**Prepared under Contract**
**No. 5-CA-81-05760**

Concrete and Structural Branch
Division of Research and Laboratory Services
Engineering and Research Center
Denver, Colorado

December 1985

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

## ACKNOWLEDGMENTS

This study was conducted by members of the Concrete and Structural Branch. Significant contributions to the report were made by C. A. Bechtold, Applied Sciences Branch, who performed the petrographic examinations, and by members of the Geotechnical Branch, who performed the shear and sliding friction tests.

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The research covered by this report was funded under the Bureau of Reclamation PRESS (Program Related Engineering and Scientific Studies) allocation No. DB-14, Longtime Concrete Studies.

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Frontispiece. -- American Falls Replacement Dam.

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## INTRODUCTION

American Falls Dam is located on the Snake River about 21 mi (34 km) southwest of Pocatello, Idaho, and about 1 mi (1.6 km) west of the town of American Falls, Idaho. The original American Falls Dam was built by the USBR (Bureau of Reclamation) and completed in 1927. This structure developed severe concrete cracking from an expansive reaction between reactive aggregates and alkalis in the cement. This cracking plus bond failures at construction joints led to the construction of a new replacement dam just downstream of the old dam.

The replacement dam, completed in 1978, was designed and constructed by Bechtel Corporation under a contract administered by the American Falls Reservoir District. After completion, it was turned over to the USBR for operation. The replacement dam has a concrete gravity center section with embankment sections extending from both ends. The new concrete gravity center section is 577 ft (176 m) long and has a structural height of 103.5 ft (31.5 m).

As part of the USBR's ongoing program to monitor and evaluate the strength and elastic properties of in-place concrete in major dams, a core test program was initiated. This program will determine the strength across horizontal construction joints and establish a data base of mechanical properties of the concrete from which changes in the concrete can be measured when future cores are extracted.

Ten-inch-diameter (254-mm) cores were extracted in January and February 1978, when the concrete was approximately 1 year old. Three of these cores were taken from the drainage gallery at elevation 4295.0 ft (1309.1 m), five were taken from the floor of the gate gallery at elevation 4300.5 ft (1310.8 m), and two were taken from a gallery at elevation 4310.0 ft (1313.7 m). Properties tested included compressive strength, tensile strength, modulus of elasticity, Poisson's ratio, and density; selected samples of the cores were also examined petrographically.

When the concrete was 5 years old, 10 more cores were drilled at locations adjacent to the 1-yr cores. The testing programs were the same except that tensile tests on the 1-yr-old concrete were made on specimens without joints, and tests on the 5-yr-old concrete were made on specimens with horizontal construction joints. Two 5-yr cores that had separated at the construction joint during drilling were tested for sliding friction.

The interior mass concrete for the dam was designed so that 80 percent of the 28-day control test cylinders would have a compressive strength in excess of 2000 lb/in$^2$ (13.8 MPa), as determined on 6- by 12-in (152- by 304-mm) cylinders after wet-screening the concrete to 1½-in

(38-mm) maximum size aggregate. The interior mass concrete contained 3-in (76-mm) maximum size aggregate and relatively low cement and pozzolan contents. Most of the cores were taken from this interior mass concrete, although it cannot be definitely established from the construction records that all of the cores were from mass interior concrete. The exterior concrete had a higher strength, and a smaller maximum size aggregate was used in some reinforced concrete for the gallery.

Aggregate supplied for the concrete came from a natural deposit. Particles were subrounded to subangular in shape and had a maximum size of 3 in (76 mm). The aggregate contained some glassy volcanics and chert, which are considered potentially alkali-reactive rock types. Further description of the aggregate is contained in appendix A.

Portland cement used in the concrete was a low-alkali type II, with an added restriction that the alkali content not exceed 0.40 percent. This precaution was taken because of the distress caused by alkali-aggregate reaction in the original American Falls Dam. Pozzolan was used in the interior mass concrete in the amount of 20 percent by mass of cementitious materials. This was a natural pozzolan ground to a high fineness and marketed under the name of Lassenite. AEA (air-entraining admixtures) and WRA (water-reducing admixtures) were also used in the concrete mix.

## CONCLUSIONS

1. An apparent decrease of 17 percent in the compressive strength of the concrete between 1 and 5 years (table 1) is attributed to the cores being tested under different moisture conditions, not to a deterioration of the concrete.

2. The elastic properties of the concrete are normal for mass concrete of this strength and age.

3. The strength of the concrete tested in direct tension is only 3.5 percent of the compressive strength. The normal expected range for mass concrete is 4 to 6 percent of the compressive strength.

4. The bond strength across the horizontal construction joints is less than the tensile strength of the concrete.

Table 1. – Properties of concrete cores, American Falls Replacement Dam.

| Core No.[1] | Compressive strength | | Modulus of elasticity | | Poisson's ratio | Density | |
|---|---|---|---|---|---|---|---|
| | lb/in² | MPa | lb/in² | GPA | | lb/ft³ | kg/m³ |
| | | | 1-yr Cores | | | | |
| 10-6-1 | 4570 | 31.5 | [2] – | [2] – | – | 152 | 2440 |
| 10-6-2 | 4120 | 28.4 | [2] – | [2] – | – | 148 | 2370 |
| 10-6-3 | 6210 | 42.8 | [2] – | [2] – | – | 151 | 2420 |
| 10-6-4 | 4790 | 33.0 | [3]6.68 | [3]46.1 | 0.17 | 153 | 2450 |
| 3-7-1 | 3080 | 21.2 | 4.73 | 32.6 | .15 | 148 | 2370 |
| 3-7-2 | 4280 | 29.5 | 4.55 | 31.3 | .15 | 149 | 2390 |
| 4-8-1 | 3520 | 24.3 | 4.19 | 28.9 | .16 | 146 | 2340 |
| 4-8-2 | 3980 | 27.4 | 4.97 | 34.3 | .13 | 149 | 2390 |
| 6-9-1 | 3710 | 25.6 | 4.30 | 29.6 | .14 | 144 | 2310 |
| 6-9-2 | 4820 | 33.2 | 3.98 | 27 4 | .12 | 146 | 2340 |
| 8-10-1 | 4380 | 30.2 | 3.86 | 26.6 | .18 | 146 | 2340 |
| 8-10-2 | 4860 | 33.5 | 4.29 | 29.6 | .15 | 146 | 2340 |
| Average | 4360 | 30.1 | 4.36 | 30.0 | 0.15 | 148 | 2370 |
| | | | 5-yr Cores | | | | |
| [4]1-1-1 | 11450 | 78.9 | 5.80 | 40 0 | 0.09 | 151 | 2420 |
| 2-2-1 | 3100 | 21.4 | 3.96 | 27.3 | .16 | 146 | 2340 |
| 7-3-1 | 361C | 24.9 | 4.09 | 28.2 | .13 | 148 | 2370 |
| 7-4-1 | 5120 | 35.3 | 4.61 | 31.8 | .14 | 145 | 2320 |
| 9-5-1 | 3050 | 21.0 | 4.87 | 33.6 | .13 | 149 | 2390 |
| 10-6-1 | 3720 | 25.6 | 4.23 | 29.2 | .12 | 148 | 2370 |
| 3-7-1 | 3080 | 21.2 | 3.73 | 25.7 | .13 | 146 | 2340 |
| 3-7-2 | 3990 | 27.5 | 4.34 | 30.1 | .11 | 148 | 2370 |
| 4-8-1 | 2970 | 20.5 | 4.49 | 31.0 | .12 | 146 | 2340 |
| 4-8-2 | 3210 | 22.1 | 4.03 | 27.8 | .12 | 148 | 2370 |
| 6-9-2 | 3650 | 25.2 | 3.97 | 27.4 | .13 | 148 | 2370 |
| 8-10-1 | 4160 | 28.7 | 3.65 | 25.2 | .11 | 145 | 2320 |
| Average | 3610 | 24.9 | 4.18 | 28.8 | 0.13 | 147 | 2350 |

[1] Specimen identification: first number is block number; second number is hole number; third number is specimen number starting at the top of the hole.
[2] Core too short for elasticity frame.
[3] Not included in averages.
[4] Specimen 1-1-1 is not included in averages.

## CORE HANDLING AND TESTING PROCEDURES

When the ten 1-yr cores extracted in 1978 arrived in the laboratory, they were logged and photographed. However, manpower shortage and higher priority work precluded immediate testing, and the cores were returned to their shipping crates and stored for 16 months. The cores were not stored in a moist environment, and they were not resaturated before testing. (A typical

1-yr core is shown on fig. 1.) There was also a delay of several months between arrival and testing of the ten 5-yr cores, but they were resaturated before testing and tested in a saturated condition.

Most cores tested for compressive strength, modulus of elasticity, and Poisson's ratio were sawed to lengths of 20 in (508 mm). However, a few specimens tested for compressive strength were shorter than 20 in; their test values were therefore corrected to an equivalent length-to-diameter ratio of 2, in accordance with ASTM C 42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" [1]*. The ends of the cores were lapped flat to a tolerance of 0.002 in (0.05 mm). Modulus of elasticity and Poisson's ratio were determined in accordance with ASTM C 469, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression" [1]. A compressometer-extensometer frame with dial gauges for measuring longitudinal and lateral deformations was placed around the specimen. A preload of 1000 lb/in$^2$ (6.9 MPa) was applied to the specimen to seat and initiate operation of the gauges. Two load sequences were then applied from 0 to 1000 lb/in$^2$ (7.0 MPa) with stress and strain values being recorded at zero load, 100 lb/in$^2$ (0.7 MPa), and 1000 lb/in$^2$ (7.0 MPa). The modulus of elasticity and Poisson's ratio were calculated from the average of the two tests between the loads of 100 and 1000 lb/in$^2$. The elasticity frame was then removed and the core loaded to failure in compression.

The cores tested in direct tension were also sawed to lengths of 20 in. All the 5-yr cores tested in tension contained a construction joint and were sawed with the construction joint at approximately midlength. The 1-yr cores were from monolithic concrete and contained no construction joints. Double end plates, 4½ in (114 mm) thick and designed to minimize deformation, were bonded to each end of the specimen with epoxy adhesive. The epoxy was cured for 24 h, during which the specimen was sealed in plastic to prevent loss of moisture. After the epoxy curing period, the specimens were placed in a hydraulic testing machine with the end plates fastened to

* Numbers in brackets refer to entries in the bibliography.

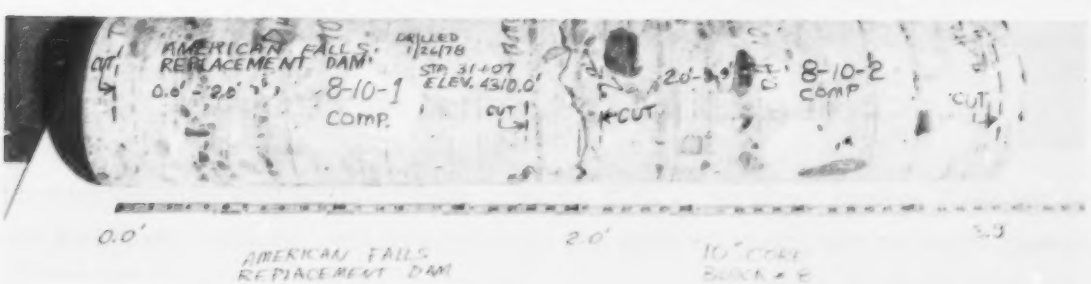


Figure 1. – Typical 10-in-diameter concrete core, American Falls Replacement Dam.

the upper and lower platens with clevis connectors. The specimens were then loaded to failure in tension.

Selected representative core fragments were given a petrographic examination. The concrete was examined megascopically, microscopically, by X-ray diffraction and differential thermal analysis, and by some qualitative physical and chemical tests. The petrographic reports for the 1- and 5-yr cores are included as appendix A and appendix B, respectively.

Two cores that had separated at the construction joint during drilling were tested for sliding friction. Increasing normal loads were applied to the specimen and the shear stress required to displace the surface at each normal load was measured. Because two tests are not adequate for statistical significance, the results of these tests are included in appendix C for record purposes only; no conclusions are made.

## TEST RESULTS

### Compressive Strength

The compressive strength data in table 1 indicate an apparent decrease in compressive strength of 17 percent between 1 and 5 years. This loss is believed to be an apparent loss and not a true loss in strength because the two sets of cores were tested in different moisture conditions. Because the 1-yr cores were not kept in a moist environment nor resaturated before testing, they were tested in a relatively dry condition. The 5-yr cores, however, were resaturated before testing and tested in a saturated condition. Because dry concrete tests 20 to 30 percent higher in compressive strength than saturated companion concrete [2], the 17-percent decrease in compressive strength does not indicate a deterioration of the concrete, but only a change in the moisture condition at the time of the tests.

ASTM C 42, "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," [1] states that test specimens should be submerged in lime-saturated water for at least 40 h immediately before testing. However, the test method also states that cores may be tested under other moisture conditions. In retrospect, it would have been preferable if the usual soaking procedure had been followed for the 1-yr cores.

Core No. 1-1-1 showed a compressive strength of 11 450 lb/in$^2$ (78.9 MPa). This is three times the average strength [3610 lb/in$^2$ (24.9 MPa)] of the other cores tested at 5 years. This abnormally

high strength may be the result of an error in testing or an error in this particular batch of concrete. Because it is not typical of the mass interior concrete, core No. 1-1-1 has not been included in the averages.

**Modulus of Elasticity and Poisson's Ratio**

The modulus of elasticity of the concrete was slightly less at 5 years [4.18 $\times$ $10^6$ lb/in$^2$ (28.8 GPa)] than at 1 year [4.36 $\times$ $10^6$ lb/in$^2$ (30.1 GPa)]. Poisson's ratio showed the same small decrease: 0.15 at 1 year and 0.13 at 5 years. These elastic properties are considered normal for mass concrete of this strength and age [3].

**Tensile Strength**

The tensile strength of the concrete and the strength of the bond across horizontal construction joints were less than expected. In the 1-yr core program, five specimens of monolithic concrete without horizontal construction joints were tested in direct tension and had an average tensile strength of 130 lb/in$^2$ (880 kPa). This strength was only 3 percent of the compressive strength. Tensile strengths of mass concrete from other USBR dams are usually in the range of 4 to 6 percent of the compressive strength. However, one specimen failed at only 20 lb/in$^2$ (140 kPa), indicating a weak, poorly consolidated area in this specimen. Yet, discounting this specimen as not representative, the average tensile strength of the remaining specimens is still only 150 lb/in$^2$ (1050 kPa), or 3.5 percent of the compressive strength.

In the 5-yr core program, four specimens containing construction joints were tested in tension. Two other specimens scheduled for tension tests separated at the joint during drilling. Of the four specimens tested, three failed at the joint. These four specimens had an average tensile strength of 120 lb/in$^2$ (840 kPa), or 3.3 percent of the average 5-yr compressive strength. Figure 2 shows a tensile test specimen that failed at the construction joint during the test.

**Petrographic Examination**

The petrographic examination indicated that the concrete was hard, well-compacted, well-hydrated and, therefore, of satisfactory quality. A comparison between the 1- and 5-yr cores examined indicates little or no change in the amounts or types of hydration products detected by X-ray diffraction analysis, differential thermal analysis, or polished surface techniques. This supports the conclusion that the change in compressive strength was caused by a change in the moisture condition at the time of the tests, and not by a deterioration of the concrete.

Figure 2. – Concrete core after failure in tensile test. Failure occurred at construction joint.

The concrete generally broke around rather than through aggregate particles. This indicated an inferior bond between the paste and the aggregate particles. The low bond strength is also indicated by the low strengths of the concrete when tested in direct tension (table 2).

Although the original American Falls Dam was weakened by alkali-aggregate reaction, no reaction with aggregate particles was observed in the 5-yr-old concrete. The combination of very low alkali cement (0.40 percent as $Na_2O$) and a high quality natural pozzolan are effectively controlling any potential for reactive expansion.

The petrographic report of the examination of samples of 1-yr cores is included as appendix A of this report, and the report on the 5-yr cores is included as appendix B.

Table 2. – Tensile strength of concrete, American Falls Replacement Dam.

| Specimen No.[1] | 1-yr cores | | 5-yr cores | | Remarks |
|---|---|---|---|---|---|
| | lb/in² | kPa | lb/in² | kPa | |
| 1-1-1 | 180 | 1240 | – | – | |
| 1-1-2 | – | – | – | – | Broke at construction joint during drilling |
| 2-2-1 | 120 | 830 | – | – | |
| 2-2-2 | – | – | 140 | 970 | |
| 7-3-1 | 20 | 140 | – | – | |
| 7-3-2 | – | – | – | – | Broke at construction joint during drilling |
| 7-4-1 | 200 | 1380 | – | – | |
| 7-4-2 | – | – | 130 | 900 | Failed at construction joint |
| 9-5-1 | 110 | 760 | – | – | |
| 9-5-2 | – | – | 70 | 480 | Failed at construction joint |
| 10-6-1 | – | – | 150 | 1030 | Failed at construction joint |
| Average | 130 | 870 | 120 | 840 | |

[1] Specimen identification: first number is block number; second number is hole number; third number is specimen number starting at the top of the hole.

**Direct Shear Tests**

Linear regression analysis of the data for the two specimens tested in direct shear for sliding friction indicated cohesion values of 1 and 33 lb/in² (7 and 228 kPa) and angles of friction of 39 and 50 degrees. A peak shearing stress of 36 lb/in² (248 kPa) was developed at a horizontal displacement of 0.1409 in (3.579 mm) for specimen AFD-4.6, tested under 50 lb/in² (345 kPa) normal stress, although about 75 percent of the maximum shearing stress was developed at the much lower horizontal displacement of 0.0184 in (0.467 mm).

Specimen AFD-4.9 required only 0.0676 in (1.72 mm) of horizontal displacement to develop the peak shearing stress of 89 lb/in² (614 kPa) under an identical 50 lb/in² normal stress. This indicates that the roughness of the surfaces in actual contact during the test contributed to higher shearing stress values for specimen AFD-4.9. A combined linear regression analysis of sliding friction data

from both specimens indicated a cohesion of 18 lb/in² (124 kPa) and a sliding friction angle of 45 degrees.

The report of the sliding friction tests is included in this report as appendix C.

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[1] Annual Book of ASTM Standards, Volume 04.02 "Concrete and Mineral Aggregates."

[2] Harboe, E. M., and Flack, H. L., "Effect of Steam Curing on Some Properties of Concrete," USBR Laboratory Report No. C-949, July 26, 1960.

[3] Harboe, E. M., "Properties of Mass Concrete in Bureau of Reclamation Dams," USBR Laboratory Report No. C-1009, December 6, 1961.

# APPENDIX A

# PETROGRAPHIC EXAMINATION OF CONCRETE CORES, 1 YEAR

OPTIONAL FORM NO. 10
JULY 1973 EDITION
GSA FPMR (41 CFR) 101-11.6

UNITED STATES GOVERNMENT

# *Memorandum*

TO : Memorandum
Chief, Concrete and Structural Branch 3/12

Denver, Colorado
DATE March 4, 1980

FROM : Chief, Applied Sciences Branch

SUBJECT: Petrographic Examination of Concrete Core - American Falls Replacement Dam - Minidoka Project, Idaho

Petrographic examination by: C. A. Bechtold

Petrographic referral code: 80-14

## MATERIAL AND METHOD OF STUDY

American Falls Replacement Dam concrete drill cores, 0.25 m (10 inch) diameter, were given a cursory examination in the Concrete Laboratory to select fragments to be further examined and tested in the Petrographic Laboratory. The cores were examined megascopically, microscopically, by XRD (X-ray diffraction) and DTA (differential thermal analysis), and by other physical and chemical tests.

The purpose of the examination is to provide petrographic data for the long-term monitoring of American Falls Replacement Dam concrete.

## PETROGRAPHIC EXAMINATION

### Cursory Examination

Drilling corrosion of the cement paste was not observed. Neither poor quality porous concrete nor a concentration of secondary deposits near the surface were observed, which suggests little if any environmental effects such as freeze-thaw deterioration.

### Petrographic Laboratory Examination

The coarse gravel particles were subrounded to subangular in shape with some calcareous coatings and a few flat and/or elongated particles. The sand was angular to subangular in shape. The sand and gravel were composed primarily of quartzose sandstone, quartzite, granite, and schist, and lesser amounts of glassy and altered volcanics, chert, obsidian and siltstone. The sand also contained increasing amounts of monomineralic grains of quartz, feldspar, amphibole, mica, and a few miscellaneous minerals. Petrographically, the aggregate would probably be of fair to satisfactory physical quality for use in concrete. Alkali-reactive rock types (glassy volcanics and chert) were probably present in sufficient quantities to be considered potentially deleteriously reactive with high-alkali cement.

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The top 10 mm of drill hole 1-1-1 contained a sand mortar mix containing mostly quartz, chert, and obsidian particles.

The concrete paste was generally reddish gray to pinkish white. The concrete broke with a hard hammer blow generally around but occasionally through some aggregate particles indicating a somewhat inferior bond between particles and paste. The gravel, sand, and paste were generally well distributed; however, the voids were somewhat variable in distribution and size with the larger voids occasionally clustered together. Several long channel voids were observed both below and around aggregate particles. Some unhydrated clinker was detected near the top surface of the core. A few shallow discontinuous fractures were present on the top surface.

Secondary products observed in the examined concrete included some calcium carbonate and a trace of ettringite which occurred in very minor amounts as minute crystal clusters in voids near the top surface of the concrete. Most voids were free of secondary deposits. Calcium carbonate was only occasionally found in the paste, which suggests little deterioration of the concrete due to environmental effects such as freeze-thaw.

No alkali silica gel or evidence of alkali aggregate reaction was detected. A fragment of concrete soaked in Denver tapwater several days showed no gel development. No evidence for sulfate attack was observed.

Portlandite and calcium silicates, including alite and belite, are normal constituents of hydrated cement. Portlandite, $Ca(OH)_2$, detected by XRD and DTA, and calcium silicates, detected by XRD, were present in normal amounts for portland cement concrete.

Water of hydration, detected by DTA, was similar to that reported in the previous memorandum and was present in normal amounts for portland cement concrete.

A few discontinuous, occasionally polyhedral microfractures were present in thin sections and polished surfaces which possibly suggests minor shrinkage due to drying.

## PETROGRAPHIC SUMMARY AND CONCLUSIONS

The petrographic examination of the portland cement concrete core revealed the following:

Petrographic evidence for freeze-thaw deterioration was minimal in the examined concrete.

The sand and gravel would probably be considered of fair to satisfactory physical quality and potentially deleteriously reactive with high-alkali cement.

The bond between paste and aggregate particles was somewhat inferior.

The paste, sand, and gravel were generally well distributed.

The voids were somewhat variable in distribution and size and some channel voids were observed.

Possible evidence of minor shrinkage was observed.

No petrographic evidence for sulfate attack or alkali aggregate reaction was detected.

Portlandite and calcium silicates were present in normal amounts for portland cement concrete.

The concrete was generally well hydrated, although some unhydrated clinker was observed near the surface of the core. The presence of unhydrated clinker near the surface might indicate poor curing or coarse cement particles; however, it was found in only one core and does not appear to be widespread or present in sufficient amounts to be interpreted as more than a local situation.

In conclusion, the examined concrete is petrographically of generally good quality as evidenced by physically fair to satisfactory aggregate, generally chemically innocous components, only a somewhat inferior bond between paste and aggregate, a lack of secondary deposits, and generally well-hydrated cement.

L. O. Tumblin Jr.

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D-1523

# APPENDIX B

## PETROGRAPHIC EXAMINATION OF CONCRETE CORES, 5 YEARS

OPTIONAL FORM NO. 10
JULY 1973 EDITION
GSA FPMR (41 CFR) 101-11.6

UNITED STATES GOVERNMENT

# *Memorandum*

TO : Memorandum
Chief, Concrete and Structural Branch

Denver, Colorado
DATE: December 15, 1983

FROM : Head, Chemistry, Petrography, and Chemical Engineering Section

SUBJECT: Petrographic Examination of Concrete Core - American Falls Dam - DB-14 Long-Time Concrete Studies - Minidoka Project, Idaho

Examined by: C. A. Bechtold

Petrographic referral code: 83-97

## INTRODUCTION

Concrete core samples (235 mm in diameter) were given a cursory examination in the Concrete Laboratory to select representative fragments to be further examined and tested in the Petrographic Laboratory. The examination was requested to determine the petrographic quality of the concrete as a part of a 5-year study program.

## PETROGRAPHIC EXAMINATION

The cores were examined megascopically, microscopically, by X-ray diffraction and differential thermal analyses, and by some qualitative physical and chemical tests.

Detailed "Petrographic Examination of Concrete" sheet is attached which includes the cursory observations and petrographic descriptions of the aggregate, paste, voids, secondary and hydration products, and fractures.

## CONCLUSIONS

The examined samples from American Falls Dam are petrographically of satisfactory quality as indicated by hard, well-compacted, and well-hydrated concrete.

There is no evidence of freeze-thaw damage, chemical attack, carbonation of cement minerals, or reaction with aggregate particles.

Although the concrete is quite sound, it generally breaks around rather than through aggregate particles. This feature will probably become more pronounced as the concrete ages. The concrete contains

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some channel voids and calcium hydroxide crystals concentrated around rock sockets which possibly suggests an initial high water content and may be the cause of the potentially weak paste-aggregate bond.

A comparison between concrete examined in 1979 and the currently examined samples indicates little or no change in amounts or types of hydration products detected by X-ray diffraction analysis, differential thermal analysis, or polished surface techniques.

J. E. Backstrom

Attachments

Copy to: D-915
D-1511
D-1523B
D-3300
D-[illegible]

CABecthold: jmw-s

PETROGRAPHIC EXAMINATION OF CONCRETE

Subject: American Falls Dam - DB-14 Long-Time Concrete Studies - Minidoka Project, Idaho

Field No.: Not given

Cursory observations: 235-mm-diameter core; generally moderately well compacted; gray-pink paste; no secondary deposits observed; paste, aggregate, and voids generally well distributed; no freeze-thaw damage observed.

Petrographic examination:

Aggregate - Gravel: subrounded in shape, occasionally flattened; consists of quartzite, quartzose sandstone, siltstone, limestone, basalt, schist, and a few obsidian and chert particles

- Sand: subangular to angular in shape; includes same rock types found in gravel as well as monomineralic grains of quartz, feldspar, amphibole, mica, garnet, magnetite, and a few miscellaneous detrital minerals

- Gravel and sand: petrographically of satisfactory to fair physical quality and not potentially deleteriously reactive with high-alkali cement

Paste - gray-pink; contains Lassenite pozzolan; generally vitreous; moderately to slightly absorptive; breaks with moderate to hard hammer blow through to generally around aggregate particles indicating a strong to potentially weak paste-aggregate bond; generally well distributed with aggregate and voids; generally moderately well compacted; slight to no effervescence with dilute hydrochloric acid

Voids - numerous small entrained air voids; some large entrapped air, water, and channel voids; generally unfilled

Secondary products - minor calcium carbonate on surface of core; voids and rock sockets generally free of secondary deposits; no silica gel developed in concrete soaked in Denver tapwater for several weeks; no soluble sulfate or chloride ions chemically detectable

Hydration products - normal amounts of calcium silicates, calcium hydroxide, and water of hydration; minor unhydrated cement particles present; some calcium hydroxide crystals concentrated around rock sockets

Fractures - generally unfractured; one tight fracture observed; no microfractures detected

# APPENDIX C

## DIRECT SHEAR TESTS OF CONCRETE CORES, 5 YEARS

OPTIONAL FORM NO. 10 (O/P 4-65) 5010-107
MAY 1962 EDITION
GSA GEN. REG. NO. 27

UNITED STATES GOVERNMENT

# *Memorandum*

TO : Memorandum
Head, Concrete Section

Denver, Colorado
DATE: MAR 2 1 1985

THROUGH: Chief, Concrete and Structural Branch jpp 3/21/85
Chief, Geotechncial Branch

FROM : Head, Rock Mechanics Section

SUBJECT: Direct Shear Tests of Concrete Specimens from American Falls Replacement Dam

Geotechnical Branch Reference No.: 85-32

Investigated by: R. L. Brammer, A. V. Scott, B. Harper, and M. J. Romansky

Written by: R. Kelsic and M. J. Romansky

Date of D-1511 requesting memorandum: July 25, 1984

Purpose of testing (Analyses): To evaluate concrete construction joints at American Falls Replacement Dam

Date testing completed: September 4, 1984

Preliminary transmittal of test results: September 12, 1984

## INTRODUCTION

Direct shear tests, as requested by Concrete and Structural Branch Memorandum dated July 25, 1984, were performed on two specimens from American Falls Replacement Dam. Both cores were approximately 10 inches in diameter.

## SPECIMEN CONDITION

Specimens were approximately 5 years in age and were tested for laboratory sliding friction.

The test specimens contained dry, open construction joints with no sign of water staining or deterioration of the aggregate or cement.

*Buy U.S. Savings Bonds Regularly on the Payroll Savings Plan*

## LABORATORY TESTING

Results of linear regression analyses of test data are summarized in table 1. Before and after testing photographs of the specimens, shear stress versus displacement plots, and a linear regression analysis summary sheet and plot for each specimen are shown in the appendix. A combined linear regression analysis of the sliding friction data from both specimens is summarized in table 2 and shown graphically on figure 1.

Linear regression analysis of the sliding friction data for specimens AFD-4.6 and AFD-4.9 indicated cohesion values of 1 and 33 $lb/in^2$ and angles of sliding friction of 39° and 50°, respectively. Peak shearing stress of 36 $lb/in^2$ was developed at a horizontal displacement of 0.1409 inch for specimen AFD-4.6 tested under 50-$lb/in^2$ normal stress, although about 75 percent of maximum shearing stress was developed at a much lower horizontal displacement of 0.0184 inch. Specimen AFD-4.9 required only 0.0676 inch of horizontal displacement to develop the peak shearing stress of 89 $lb/in^2$ under an identical 50-$lb/in^2$ normal stress. This indicates that roughness of the surfaces in actual contact during the test contributed to higher shearing stress values for specimen AFD-4.9. A combined linear regression analysis of sliding friction data from both specimens indicated a cohesion of 18 $lb/in^2$ and a sliding friction angle of 45°.

## CONCLUSIONS

Test results should be used with discretion because of the limited number of test specimens (two) and succeeding small number of sliding friction runs employed in this test program. Before using these test results, a thorough review of available drill logs and construction reports should be conducted to locate areas of joint contact similar to the test specimens. This is considered necessary to define where these shear strength parameters might be applicable.

F. C. Kundig

Attachment

Copy to: D-1511
D-1542
D-1543 (2)

Table 1. - Direct shear test results - Summary

| Specimen No. | Feature | Surface condition | Area ($in^2$) | Shear[1] stress ($lb/in^2$) | Cohesion ($lb/in^2$) | Friction angle (degrees) | Comments[2] |
|---|---|---|---|---|---|---|---|
| AFD-4.6 | Open joint | Smooth and flat | 70.30 | 36 | 1 | 39 | 50, 100, 151 |
| AFD-4.9 | Open joint | Smooth and wavy | 70.17 | 89 | 33 | 50 | 50, 100, 150 |

1 Peak shearing stress developed at the first applied normal stress.
2 Loading sequence (normal stress in $lb/in^2$).

Table 2. - Combined test data summary

| Feature | Break bond[1] | Sliding friction Linear regression results | |
|---|---|---|---|
| | | Angle of sliding friction (degrees) | Cohesion ($lb/in^2$) |
| American Falls Replacement Dam | | 45° | 18 |

1 No intact specimens available for testing.

Sheet 1 of 1
Project: Minidoka
American Falls Replacement Dam

Tables 1 and 2

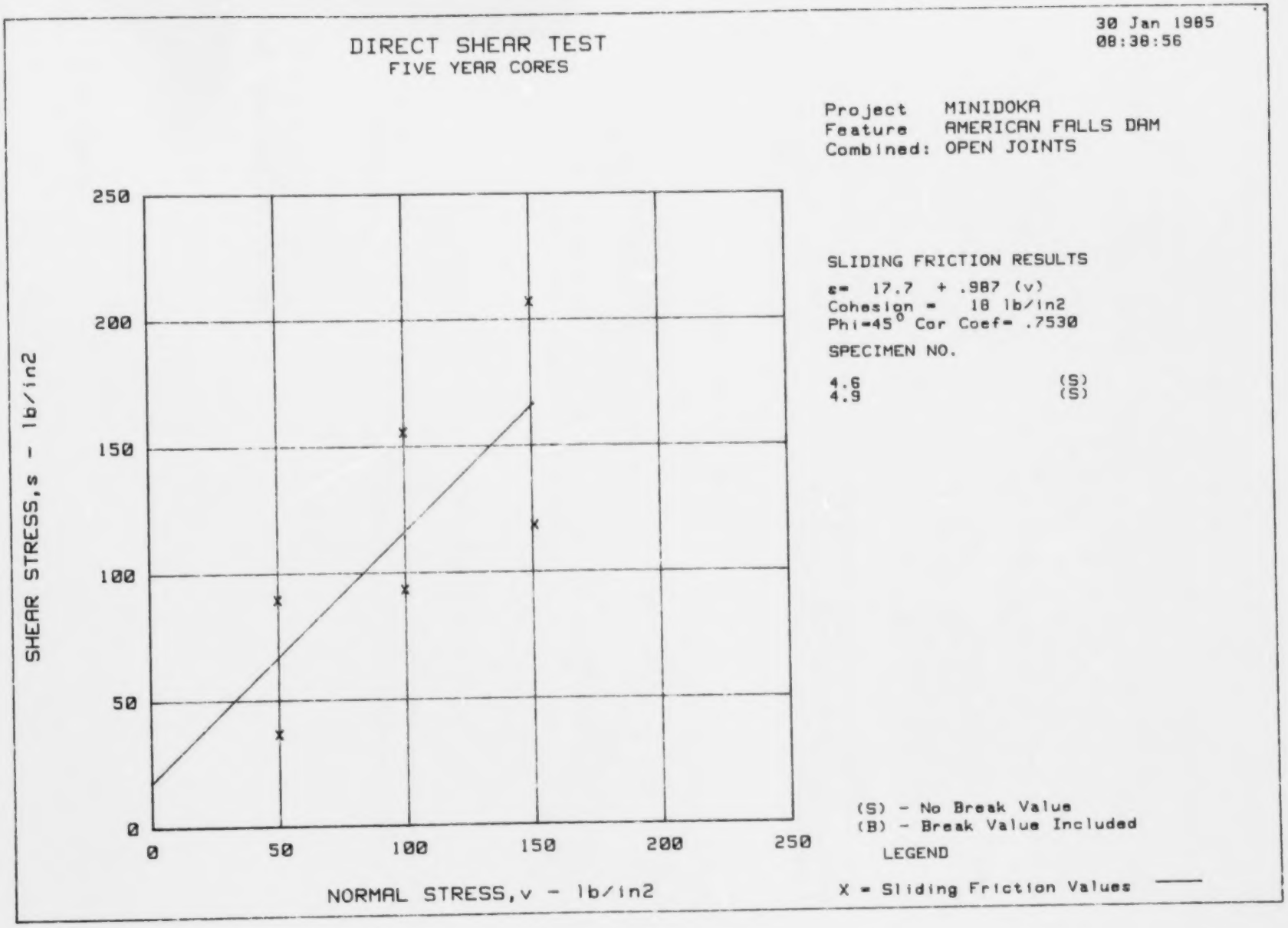

DIRECT SHEAR TEST
FIVE YEAR CORES
30 Jan 1985
08:38:56
Project MINIDOKA
Feature AMERICAN FALLS DAM
Combined: OPEN JOINTS
SLIDING FRICTION RESULTS
s= 17.7 + .987 (v)
Cohesion = 18 lb/in2
Phi=45° Cor Coef= .7530
SPECIMEN NO.
4.6 (S)
4.9 (S)
SHEAR STRESS,s - lb/in2
NORMAL STRESS,v - lb/in2
0
50
100
150
200
250
(S) - No Break Value
(B) - Break Value Included
LEGEND
X = Sliding Friction Values

Figure 1

# APPENDIX

Specimen before testing
(Parallel to sliding direction)

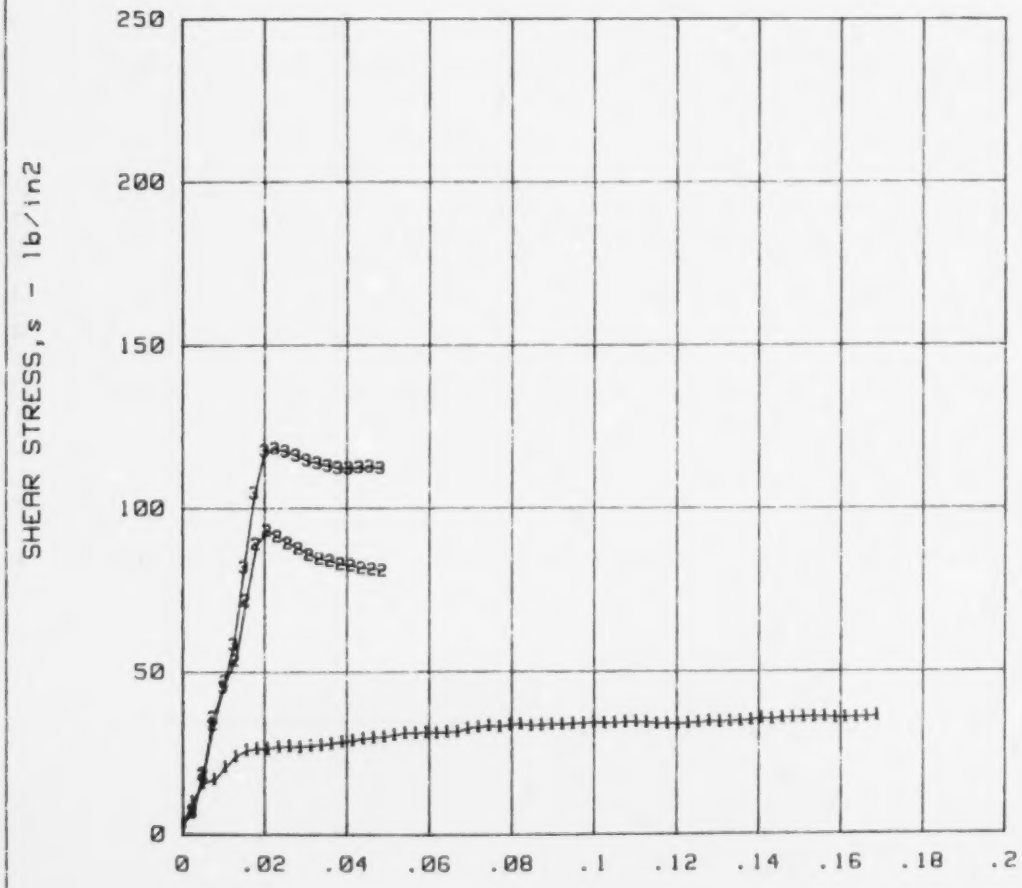
Specimen before testing
(Plan view of sliding surface)

Specimen after testing

# DIRECT SHEAR TEST

30 Jan 1985
08:14:50

| | |
|---|---|
| Project | MINIDOKA |
| Feature | AMERICAN FALLS DAM |
| Type | OPEN JOINT - 5 YEAR |
| Spec no. | 4.6 |
| Tested By: | R.L.B. |
| Date Tested | 08/24/84 |
| Area | 70.301 Sq. in. |

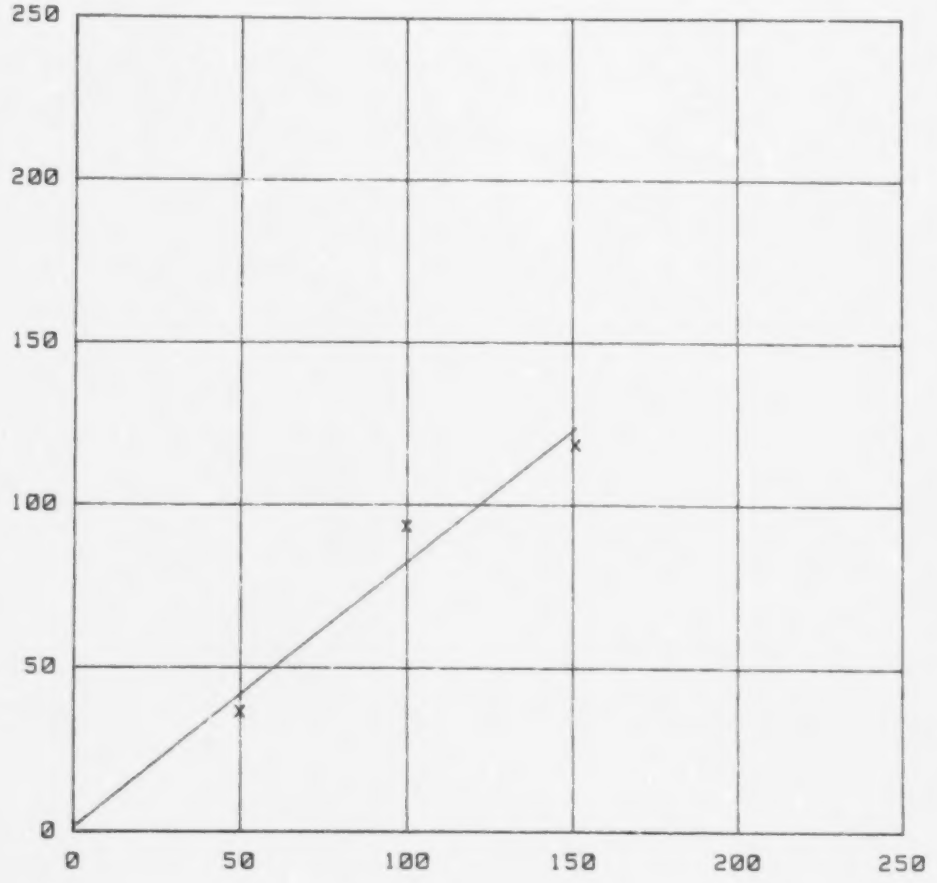



LEGEND

| NORMAL lb/in2 | SYMBOL |
|---|---|
| 50 | 1 |
| 100 | 2 |
| 151 | 3 |

# DIRECT SHEAR TEST

30 Jan 1985
08:17:32




| | |
|---|---|
| Project | MINIDOKA |
| Feature | AMERICAN FALLS DAM |
| Type | OPEN JOINT - 5 YEAR |
| Spec no. | 4.6 |
| Tested By: | R.L.B. |
| Date Tested | 08/24/84 |
| Area | 70.301 Sq. in. |

| v lb/in2 | s lb/in2 |
|---|---|
| 50 | 36 |
| 100 | 93 |
| 151 | 118 |

SLIDING FRICTION RESULTS

s= 1.4 + .810 (v)

COHESION = 1 lb/in2

PHI= 39° COR COEF= .9744

30 Jan 1985
08:19:52

DATA SUMMARY

Project MINIDOKA
Feature AMERICAN FALLS DAM
Type OPEN JOINT - 5 YEAR
Spec no. 4.6
Tested By: R.L.B.
Date Tested 08/24/84
Area 70.301 Sq. in.

| NORMAL LOAD (lbs) | SHEAR LOAD (lbs) | DISPLACEMENT (ins) | NORMAL STRESS (lb/in2) | SHEAR STRESS (lb/in2) |
|---|---|---|---|---|
| 3515 | 2558 | .1689 | 50 | 36 |
| 7030 | 6558 | .0208 | 100 | 93 |
| 10615 | 8317 | .0227 | 151 | 118 |

SUM X*X = 35301
SUM Y*Y = 24022
SUM X*Y = 29012
SUM X = 301
SUM Y = 248

SLIDING FRICTION RESULTS
s= 1.4 + .810 (v)
COHESION = 1 lb/in2
PHI= 39° COR COEF= .9744

30 Jan 1985 08:08:54

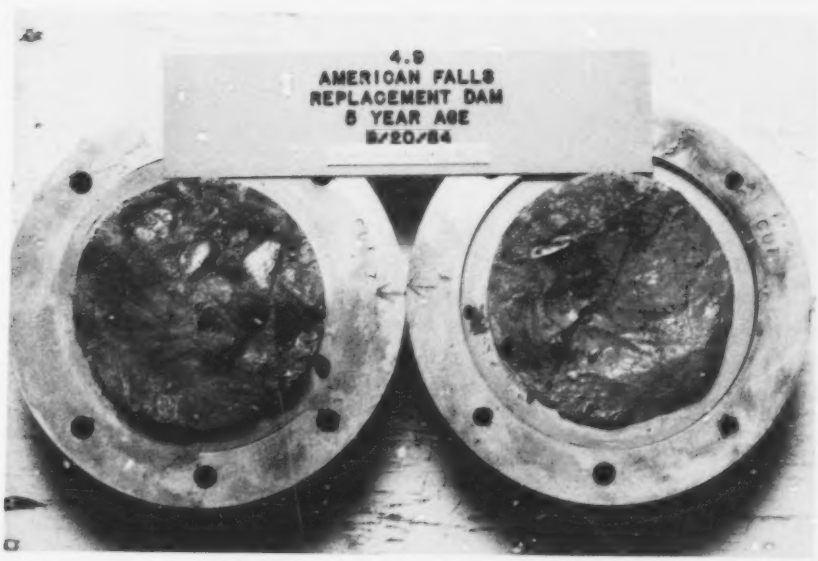
FINAL DATA

Project MINIDOKA
Feature AMERICAN FALLS DAM
Type OPEN JOINT - 5 YEAR
Spec no. 4.6
Tested By: R.L.B.
Date Tested 08/24/84
Area 70.301 Sq. in.

| NORMAL STRESS lb/in2 | SHEAR STRESS lb/in2 | DISPLACEMENT inches |
|---|---|---|
| 50 | 3 | 0.0000 |
| 50 | 10 | .0025 |
| 50 | 16 | .0051 |
| 50 | 17 | .0079 |
| 50 | 21 | .0106 |
| 50 | 24 | .0132 |
| 50 | 26 | .0158 |
| 50 | 27 | .0184 |
| 50 | 26 | .0210 |
| 50 | 27 | .0237 |
| 50 | 27 | .0263 |
| 50 | 27 | .0289 |
| 50 | 28 | .0314 |
| 50 | 28 | .0340 |
| 50 | 28 | .0365 |
| 50 | 29 | .0391 |
| 50 | 29 | .0417 |
| 50 | 30 | .0442 |
| 50 | 30 | .0467 |
| 50 | 30 | .0493 |
| 50 | 31 | .0518 |
| 50 | 31 | .0544 |
| 50 | 31 | .0569 |
| 50 | 32 | .0595 |
| 50 | 31 | .0620 |
| 50 | 32 | .0645 |
| 50 | 32 | .0670 |
| 50 | 33 | .0695 |
| 50 | 33 | .0721 |
| 50 | 34 | .0747 |
| 50 | 33 | .0773 |
| 50 | 34 | .0798 |
| 50 | 34 | .0824 |
| 50 | 34 | .0849 |
| 50 | 34 | .0874 |
| 50 | 34 | .0899 |
| 50 | 34 | .0925 |
| 50 | 34 | .0951 |
| 50 | 34 | .0976 |
| 50 | 35 | .1002 |
| 50 | 34 | .1027 |
| 50 | 34 | .1053 |
| 50 | 35 | .1078 |
| 50 | 35 | .1104 |
| 50 | 34 | .1129 |
| 50 | 34 | .1155 |
| 50 | 34 | .1180 |
| 50 | 34 | .1205 |
| 50 | 34 | .1230 |
| 50 | 34 | .1256 |
| 50 | 35 | .1282 |
| 50 | 35 | .1307 |
| 50 | 35 | .1333 |
| 50 | 35 | .1358 |
| 50 | 35 | .1383 |
| 50 | 36 | .1409 |
| 50 | 35 | .1434 |
| 50 | 36 | .1460 |
| 50 | 36 | .1485 |
| 50 | 36 | .1511 |
| 50 | 36 | .1537 |
| 50 | 36 | .1562 |
| 50 | 36 | .1588 |
| 50 | 36 | .1613 |
| 50 | 36 | .1639 |
| 50 | 36 | .1664 |
| -50 | 36 | .1689 |
| 100 | 4 | 0.0000 |
| 101 | 7 | .0026 |
| 101 | 19 | .0051 |
| 102 | 34 | .0077 |
| 102 | 47 | .0104 |
| 101 | 53 | .0130 |
| 100 | 72 | .0156 |
| 100 | 89 | .0181 |
| 100 | 93 | .0208 |
| 100 | 91 | .0234 |
| 100 | 89 | .0259 |
| 100 | 88 | .0284 |
| 99 | 86 | .0310 |
| 99 | 85 | .0336 |
| 100 | 84 | .0361 |
| 100 | 83 | .0387 |
| 100 | 83 | .0412 |
| 100 | 82 | .0437 |
| 100 | 81 | .0463 |
| -101 | 81 | .0488 |
| 150 | 3 | 0.0000 |
| 150 | 7 | .0025 |
| 150 | 18 | .0050 |
| 151 | 36 | .0076 |
| 151 | 45 | .0101 |
| 150 | 58 | .0126 |
| 151 | 82 | .0151 |
| 152 | 105 | .0177 |
| 152 | 118 | .0202 |
| 151 | 118 | .0227 |
| 150 | 117 | .0253 |
| 150 | 116 | .0279 |
| 150 | 115 | .0304 |
| 149 | 114 | .0330 |
| 149 | 113 | .0355 |
| 149 | 112 | .0381 |
| 149 | 112 | .0406 |
| 149 | 112 | .0432 |
| 149 | 113 | .0457 |
| -149 | 112 | .0483 |

Specimen before testing
(Parallel to sliding direction)

Specimen before testing
(Plan view of sliding surface)

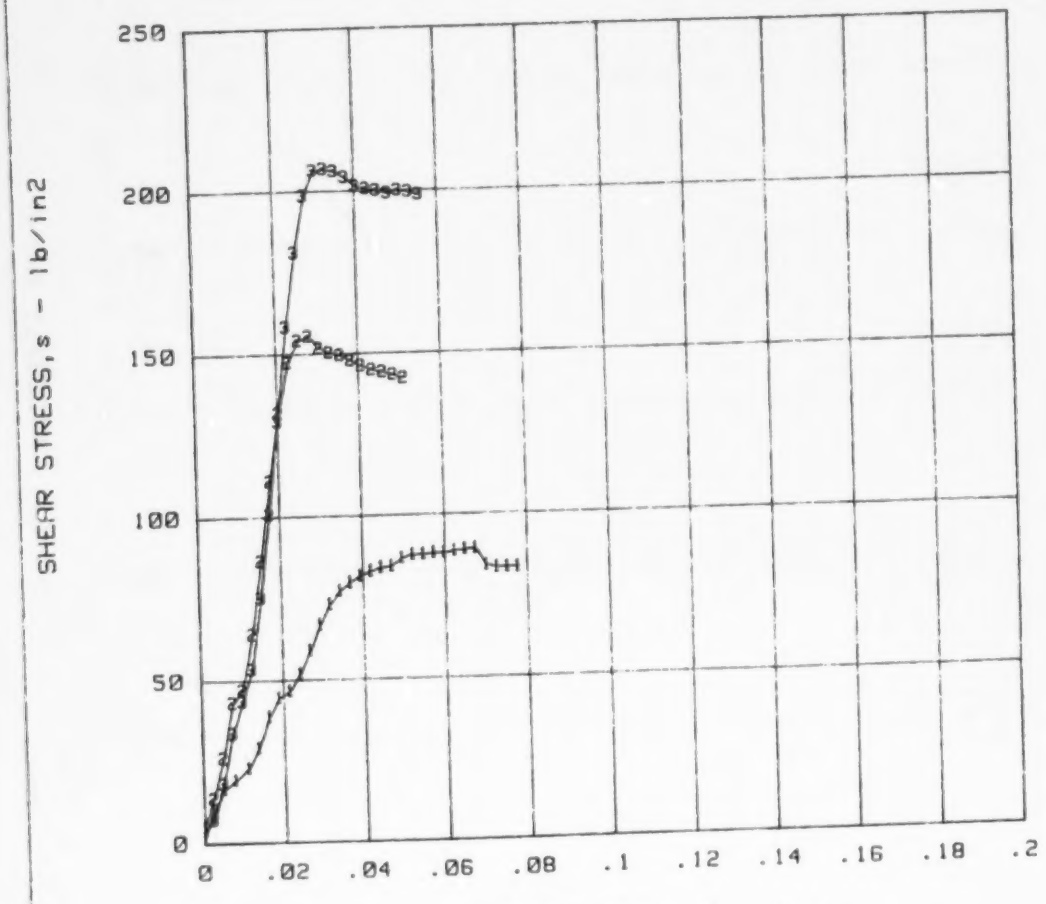



Specimen after testing

30 Jan 1985
08:25:43

# DIRECT SHEAR TEST

| | |
|---|---|
| Project | MINIDOKA |
| Feature | AMERICAN FALLS DAM |
| Type | OPEN JOINT - 5 YEAR |
| Spec no. | 4.9 |
| Tested By: | R.L.B. |
| Date Tested | 08/24/84 |
| Area | 70.168 Sq. in. |

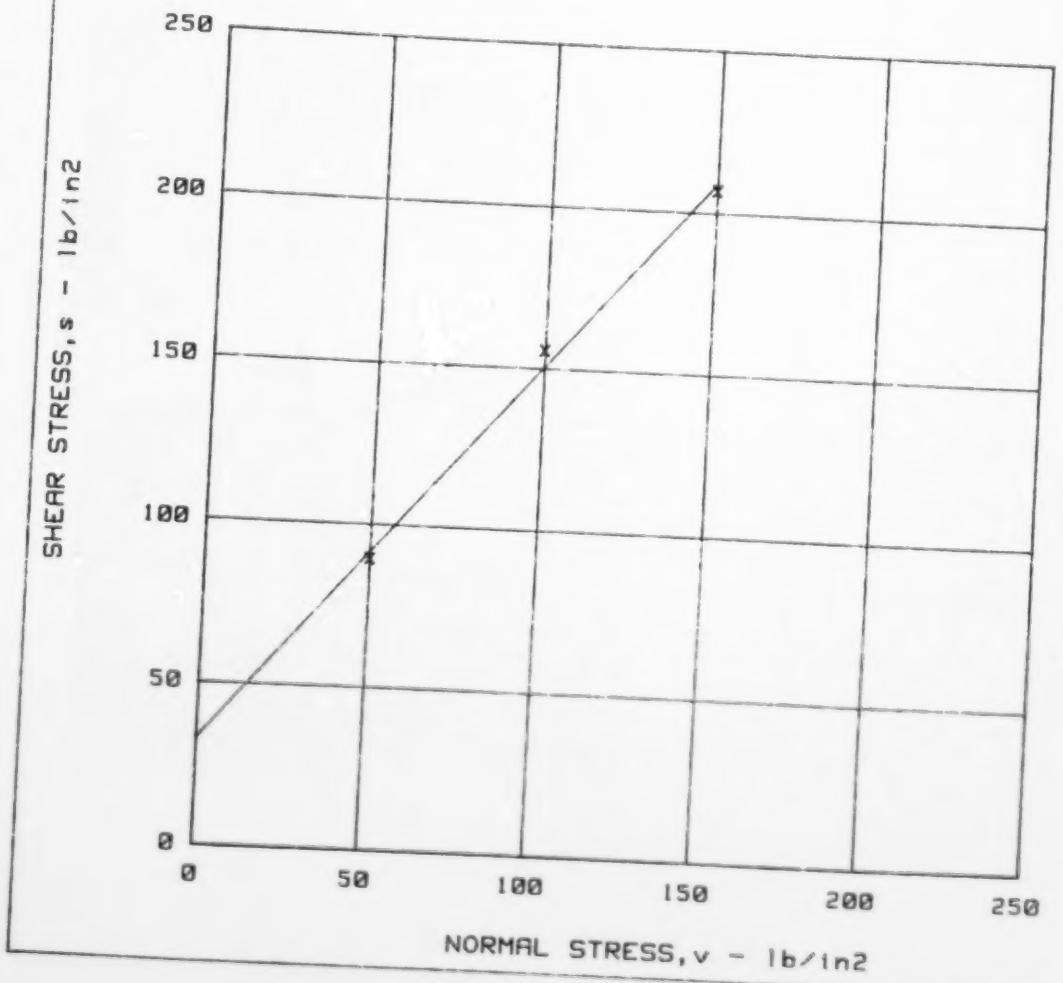



LEGEND

| NORMAL lb/in2 | SYMBOL |
|---|---|
| 50 | 1 |
| 100 | 2 |
| 150 | 3 |

# DIRECT SHEAR TEST

30 Jan 1985
08:28:18




| | |
|---|---|
| Project | MINIDOKA |
| Feature | AMERICAN FALLS DAM |
| Type | OPEN JOINT - 5 YEAR |
| Spec no. | 4.9 |
| Tested By: | R.L.B. |
| Date Tested | 08/24/84 |
| Area | 70.168 Sq. in. |

| v lb/in2 | s lb/in2 |
|---|---|
| 50 | 89 |
| 100 | 155 |
| 150 | 207 |

SLIDING FRICTION RESULTS

s= 33.0 +1.174 (v)

COHESION = 33 lb/in2

PHI= 50$^0$ COR COEF= .9974

30 Jan 1985
08:30:36

DATA SUMMARY

Project MINIDOKA
Feature AMERICAN FALLS DAM
Type OPEN JOINT - 5 YEAR
Spec no. 4.9
Tested By: R.L.B.
Date Tested 08/24/84
Area 70.168 Sq. in.

| NORMAL LOAD (lbs) | SHEAR LOAD (lbs) | DISPLACEMENT (ins) | NORMAL STRESS (lb/in2) | SHEAR STRESS (lb/in2) |
|---|---|---|---|---|
| 3508 | 6267 | .0676 | 50 | 89 |
| 7017 | 10900 | .0282 | 100 | 155 |
| 10525 | 14508 | .0328 | 150 | 207 |

SUM X*X = 35000
SUM Y*Y = 74858
SUM X*Y = 51014
SUM X = 300
SUM Y = 451

SLIDING FRICTION RESULTS
s= 33.0 +1.174 (v)
COHESION = 33 lb/in2
PHI= 50° COR COEF= .9974

30 Jan 1985 08:10:34

FINAL DATA

Project MINIDOKA
Feature AMERICAN FALLS DAM
Type OPEN JOINT - 5 YEAR
Spec no. 4.9
Tested By: R.L.B.
Date Tested 08/24/84
Area 70.168 Sq. in.

| NORMAL STRESS lb/in2 | SHEAR STRESS lb/in2 | DISPLACEMENT inches |
|---|---|---|
| 50 | 2 | 0.0000 |
| 50 | 10 | .0025 |
| 50 | 17 | .0052 |
| 50 | 19 | .0081 |
| 50 | 23 | .0113 |
| 50 | 29 | .0139 |
| 50 | 39 | .0165 |
| 50 | 45 | .0191 |
| 50 | 47 | .0217 |
| 50 | 52 | .0243 |
| 50 | 59 | .0269 |
| 50 | 67 | .0294 |
| 50 | 73 | .0319 |
| 50 | 77 | .0344 |
| 50 | 80 | .0370 |
| 50 | 82 | .0395 |
| 50 | 83 | .0421 |
| 50 | 84 | .0446 |
| 50 | 84 | .0471 |
| 50 | 87 | .0498 |
| 50 | 88 | .0523 |
| 50 | 88 | .0549 |
| 50 | 88 | .0575 |
| 51 | 88 | .0600 |
| 50 | 89 | .0625 |
| 50 | 89 | .0651 |
| 50 | 89 | .0676 |
| 50 | 84 | .0702 |
| 50 | 83 | .0727 |
| 50 | 83 | .0753 |
| -50 | 83 | .0778 |
| 100 | 5 | 0.0000 |
| 100 | 14 | .0026 |
| 101 | 26 | .0051 |
| 101 | 43 | .0077 |
| 101 | 47 | .0102 |
| 101 | 64 | .0128 |
| 100 | 86 | .0154 |
| 100 | 111 | .0180 |
| 100 | 132 | .0205 |
| 100 | 147 | .0230 |
| 100 | 154 | .0255 |
| 100 | 155 | .0282 |
| 100 | 152 | .0307 |
| 100 | 150 | .0332 |
| 100 | 149 | .0358 |
| 100 | 148 | .0384 |
| 100 | 146 | .0409 |
| 100 | 145 | .0435 |
| 100 | 144 | .0461 |
| 100 | 143 | .0486 |
| -100 | 142 | .0511 |
| 150 | 4 | 0.0000 |
| 151 | 7 | .0025 |
| 149 | 18 | .0050 |
| 150 | 34 | .0075 |
| 151 | 43 | .0101 |
| 150 | 53 | .0126 |
| 150 | 75 | .0151 |
| 150 | 101 | .0176 |
| 149 | 129 | .0202 |
| 150 | 158 | .0227 |
| 149 | 181 | .0252 |
| 150 | 199 | .0277 |
| 150 | 206 | .0302 |
| 150 | 207 | .0328 |
| 151 | 206 | .0353 |
| 151 | 204 | .0378 |
| 149 | 201 | .0404 |
| 149 | 201 | .0430 |
| 149 | 200 | .0456 |
| 149 | 199 | .0481 |
| 150 | 200 | .0507 |
| 150 | 199 | .0532 |
| -149 | 198 | .0558 |

GPO 850-125

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*Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.*

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